Rockfall and Subsidence on Mumbai-Pune Expressway

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ABSTRACT: *Mumbai-Pune, India's first expressway, which crosses the mountainous and rugged Deccan Trap Province, suffered from a large amount of rockfall and major landslides in 2003 and 2004. A significant number of accidents and fatalities have occurred on the expressway as a result of such events. On the basis of frequency and magnitude of the recorded incidences of rockfall, seventeen critical areas of the road have been deemed in need of investigations and mitigation. One of the problematic areas that suffered from rockfall and subsidence is at km 41 on the expressway, adjacent to a tunnel (Adoshi). This location has been chosen for detailed study. This area extends for 500 m along the expressway, and the embankment has subsided by 2 to 3 m along the entire length. Based on the geological, geomorphologic and geotechnical investigations, causative factors for rockfall and subsidence at this location have been established. This paper presents the field investigations, probable causes, stability analysis and remedial measures for controlling the subsidence and rock fall.*

KEYWORDS: Expressway, rockfall, subsidence, drainage, steel rope net, SFRS

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

Mumbai, the commercial capital of India, is growing significantly in size and population. Pune, the cultural capital of Maharashtra, is growing into a major industrial and commercial centre. Hence, the importance of the Mumbai-Pune road has increased tremendously in the last 8–10 years. The annual increase in traffic on the existing National Highway and other link roads results in frequent traffic jams, accidents and loss of travel time. According to one estimate, 400 people are killed due to accidents on the existing Mumbai-Pune National Highway each year. For these reasons, the government deemed it necessary to construct a new, separate expressway (Figure 1). The expressway, totaling 95 km in length, opened on March 1, 2002. It has reduced the travel time between the cities of Mumbai and Pune to approximately two hours. It starts at Kalamboli (near Panvel) and ends at Dehu Road (near Pune). The expressway handles up to 100,000 passenger carrying units (PCUs) per day. During 2003 and 2004, the expressway experienced problems with rockfall. A reconnaissance inventory carried out by the Central Road Research Institute, New Delhi, India revealed that about 90% of the slope failures along the stretch are due to rockfall and the remaining 10% are due to debris flows, subsidence and sliding. On the basis of frequency and magnitude of the recorded incidences, more than seventeen problematic rockfall-prone areas have been identified. These critical areas of rockfall are located at: Amritanjan Bridge, km 14, km 18, km 21.5, km 29.8, km 26.491, km 38.9, km 41, km 41.679, km 42.36, km 47.5, km 68 and km 70. Investigations of all the identified areas have been carried out from various aspects that were thought critical to slope stability, including geological, geomorphologic and geotechnical aspects. The extensive field investigations were carried out in an effort to understand the possible causes and mechanisms of rockfall and subsidence at the specified locations. On the basis of these investigations, appropriate remedial measures were suggested for specific locations. This paper intends to present, in brief, the field

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investigations carried out to assess the probable causes and the remedial measures recommended in order to control the rockfall and subsidence at km 41 on the Mumbai-Pune Expressway.

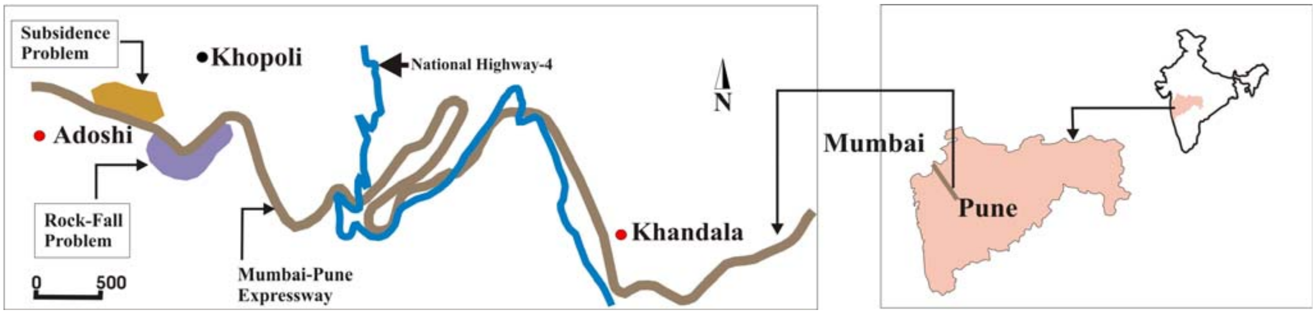


Figure 1. Location of the Mumbai-Pune expressway.

ROCKFALL AND DEBRIS FLOW

Figure 1 shows the location of the Adoshi site. Rockfall and debris flow are the predominant problems at this location. The first incident of rockfall and debris flow (Figure 2) reportedly occurred in 2003–2004, during the monsoon. Since then, rock blocks have been intermittently falling from the face and top of the slope, damaging the side drains as well as the middle of the expressway. The size of the rock blocks varies from a few centimeters to a meter—and sometimes even larger—in diameter (Figure 3). The hill slope geometry at this location is such that rock blocks fall directly onto the expressway. There is no space or slope bench on the side of the expressway and the hill; as a result, there is no reduction in the momentum of the detached rock when it plummets toward the expressway at this location. This direct fall makes the impact more dangerous and more potentially fatal when rock blocks impact a running vehicle. Such incidences are particularly frequent during the rainy season. Rockfall can be caused by a variety of reasons such as unfavorable jointing, freeze-thaw cycles, water effects, earthquake loading, etc. (Topal et al., 2007; Dorren, 2003). In the present case, on the basis of field observations and the records of failures, the jointing and the effect of water are expected to be major contributors to the failure. Rockfall can be studied using various techniques such as empirical, experimental and modeling procedures (Giani, 1992; Okura et al., 2000; Evans and Hungr, 1993).

Figure 2. General view of rockfall and debris.

Figure 3. Rockfall and debris in 2004.

GENERAL ROCK SLOPE CONDITIONS

The area is made up of basaltic rocks. Generally speaking, there are two types of basaltic rocks: compact and amygdoidal. Also, there is a layer of weak material in between the two varieties of basalt. These conditions are favourable for erosion either by surficial action or from internal erosion and piping. Such occurrences of erosion have been observed not only at this location but in many of the other trouble areas. It was observed that the weak layer underlying the rocks with open joints facilitates rapid erosion due to seepage, undermining the adjacent rocks and causing rock failure. Along the expressway, cut slopes (jointed/fractured rock mass) gradually become loose due to raveling (Goodman, 2000). The loosened rock mass comes down onto the expressway.

Changes in the slope profile may create various modes of failure such as fall, toppling, glide or slide. Moreover, velocity, weight and shape of the block and properties of slope-forming materials greatly influence events of rockfall (Giani, 1992; Azzoni et al., 1995; Dorren, 2003). In many sections of the trouble sites, wedge failure due to the intersection of nonparallel discontinuity surfaces have been observed. Large blocks of the detached rock masses slide from the slope and land on the expressway. The critical wedges typically do not fail until favorable conditions for sliding exist. Toppling failure is another type of failure that has been observed in this area. Toppling failure involves the rotation of a column or block of rock about some fixed base and the simple geometrical conditions governing the toppling of a single block on an inclined surface (Hoek and Bray, 1981). Rockfalls occur by the detachment of rock blocks from steeply dipping discontinuities along which little or no shear displacement takes place. The blocks then descend, primarily through the air, falling, bouncing or rolling at rapid (0.3 m/min) to extremely rapid (3–30 m/s) velocities.

A wide variety of methods are used today to prevent rockfall along roads, including trimming, rock bolts, shotcrete and drainage. Davis and Abdul Shakoor (2005) studied the effectiveness of catchment ditches along Ohio roadways at 100 sites using the Oregon rock fall hazard rating system. Navaratnarajah et al (2005) researched global stability, anchor spacing and support cable loads in wire mesh and cable net slope protection systems. They focused on the effect of mesh weight (the friction between mesh and rock surfaces) and the accumulation of debris on the overall stability of the systems. However, it might be too expensive or too difficult to prevent all rocks from falling. If it is not as essential to prevent the rock from falling in a certain location, it may be appropriate to allow the rocks to fall but to prevent them from entering the roadway.

FIELD INVESTIGATIONS

Geological Investigation

Field observational methods are one of the most useful and effective investigation techniques. Based on the results acquired by implementing these techniques, one can accurately evaluate the actual site conditions and make conclusions regarding the type of investigation, analysis and protective measures required. A thorough field investigation has been carried out to identify the causes and mechanism of the failures.

The ground conditions in the study area are dominated by basalt, which is an extrusive rock created by the outpouring of volcanic magma. The magma cools quickly, allowing only small crystals to form. Basaltic lava flows for great distances before solidifying. Successive eruptions of basalt have formed the Deccan plateau region of southwest India, including the current study area. The area is conspicuously uniform, consisting of series of Deccan Trap flow (Upper Cretaceous to Lower Eocene age), which are occasionally intruded by a number of basic intrusive. The basalts are mainly capped by lateritic soil.

Basalt rock is composed of basic lava that has a lower than average silica content. Basalt forms in the areas that are spreading and forms horizontal layers of lava extruded on the surface. Upon reaching the surface, the cooling of the basalt is rapid at the top and bottom due to contact with the ground surface and atmosphere, resulting in the formation of small crystals in these parts. The central part, however, cools slower than the other two and forms slightly bigger crystals. Such a condition leads to the differential weathering that affects particularly weaker zones of the rock. Successive episodes of the lava flow result in the layered arrangement of the basaltic flow. Since the composition of the volcanic flow differs in each episode, the rock mass possesses layered structures with horizontal continuity but vertical heterogeneity. Commonly, the natural hill slopes produced from this type of rock have a stepped shape, with the steeper portion corresponding to harder layers and flatter areas to the softer layers. Predominant vertical jointing favors the formation of steep slopes. In both cases, the structure of rock mass is usually of layers of different constitutions, altering weak and strong rocks, with preferential seepage along horizontal discontinuities or weak layers.

The rocks at the study site are basalts of two types: (a) amygdaloidal and (b) compact basalt. Both types of basalts are separated by a thin layer of highly disaggregated material derived from the weathering of the basaltic layer itself. In many places, this layer separates basalts of same type also. This layer at times creates one of the potential conditions for dislodgement of basaltic rocks upon weathering of the thin layers.

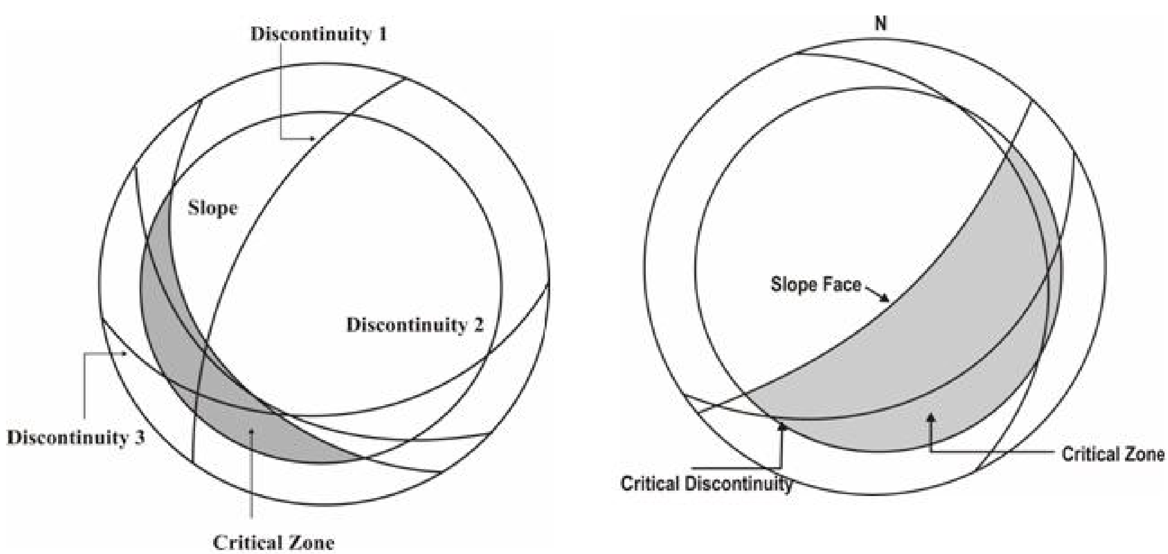


Figure 4. Markland test for planar failure on two sites showing critical zone for potential failure.

It was noticed that the thin, altered top of the basalt layer generally has seepage (or moisture) in it, which is one of the potential conditions for making the slope susceptible to fail. Generally there are several fractures and few of the joint planes. The fractures on the rocks, which are normally confused with joints, have prominent attitudes. There are mainly three major joints, but one of the most prominent planes of weakness is the vertical joint (typical of basaltic layer, i.e. columnar joint). This prominent joint is repeated at regular intervals. The stereographic projection diagram, using the field data of joints, revealed the formation of a prominent wedge at the slope face. The vertical joints intersect to form wedges on the slope independently as well as when they intersect other steeply dipping fractures and other horizontal and subhorizontal fractures, as shown in Figure 4. In most cases, the plunge of the intersection of these two joint sets lies toward NNW in the direction of the slope. Additionally, the plunge of the intersection of the joints is more than the angle of the slope most of the time. The Markland Test was used to define the critical zone of planar and wedge failure in the stereographic projection. In the case of planar failure, the dip of the discontinuity exceeds the angle of friction of the rock; the discontinuity daylights in the slope face and dips less than the slope. In the case of wedge failure, the plunge of the line of intersection of two joints is considered (Figure 4) (Markland, 1972).

Geomorphological Investigations

Geomorphologically, the slope can be divided into three parts (Figure 2): The first part, above the existing crown of the slide, extends for about 400 meters from the crown upward. This part has undulated topography of varying thicknesses of overburden consisting of reddish lateritic soil mixed with boulders of up to 1m in diameter. The general slope angle of the uphill slope ranges from 40^0–45^0.

The slope is well covered by trees. The uphill slope has good catchment, and the rainwater generally drains out through the natural streams on both flanks of the slide. A considerable quantity of water also flows down the middle of the slope, above the affected portion. Near the crown there are a number of loose stone blocks, which rest on the slope. These may facilitate blocking and infiltration of more water into the slope during rains. The crown consists of a thin soil cover of a few centimeters to about one-third of a meter in thickness, below which lies 1–2-meter thick weathered and jointed basalt. This layer of weathered and jointed basalt, along with the thin overburden, fails along a smooth sheet joint surface and falls on the expressway through the adjoining vertical cut slope (Figure 2). This vertical cut of about 40 m in height has many joint intersections separating the blocks of the basaltic layer (Figure 5). A stream, guided by the intersection of one of the oblique and sub-horizontal fractures (Figure 6), remains dry except during the monsoon months when it serves as one of the main draining streams (Figure 7). A significant amount of water drains from the above catchment area.

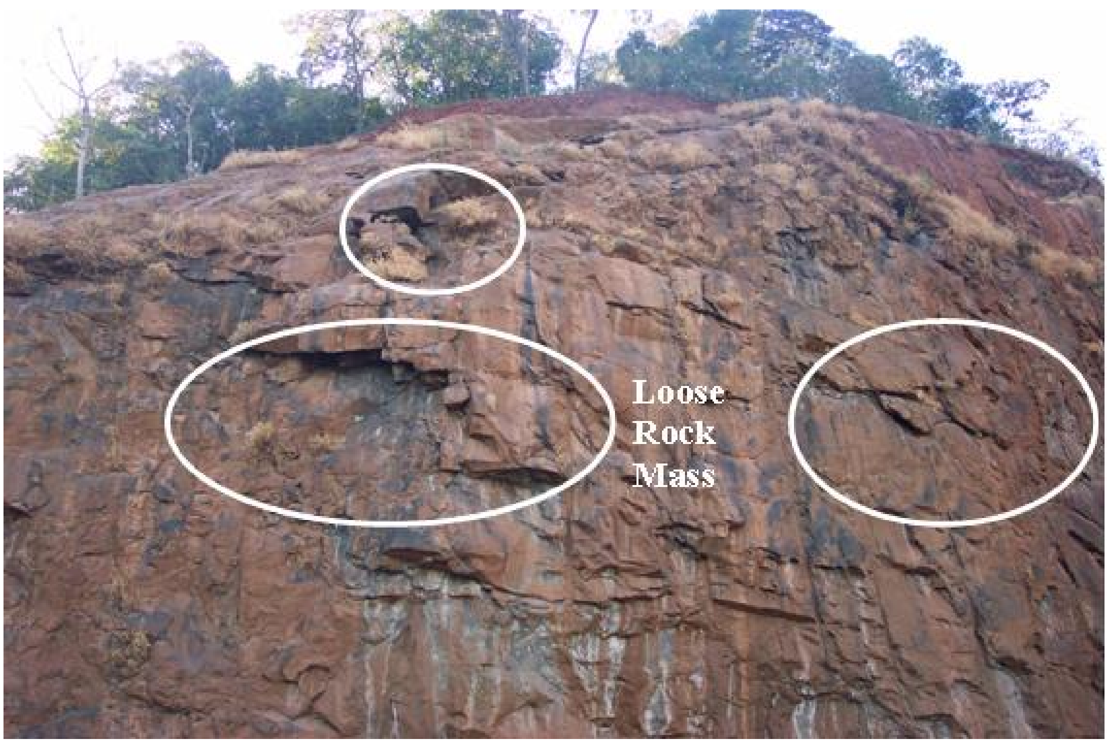


Figure 5. Vertical joints and formation of wedge along slope face.

Figure 6. Intersection of oblique and sub-horizontal fractures.

Figure 7. Significant water drainage during a monsoon.

GEOTECHNICAL INVESTIGATION

The geotechnical investigations include evaluation of the condition of the rock as well as the soil at the landslide area. A thin layer of soil ranging from a few centimeters to 1 meter in depth (and at few places even more than one meter) exists. The liquid limit of soil varies from 43% to 53%, PI of the soil varies from 15% to 28%, and in some locations the soil is nonplastic and c' is 3 kPa and $\varphi' = 30^0$.

The condition of the rock can be defined as amygdaloidal basalt with vesicules and coarse grains having unconfined strength of 150 MPa and the compact, fine-grained basalt with an unconfined strength of 180 MPa (MSRDC Report, 1995). The strength of the rocks was also estimated using a Schmidt and geological hammer according to Hoek and Brown (1997) although the strength of the intact rock does not contribute significantly in the case of fractured or jointed rocks. In these rocks, the discontinuities play a major role in forming favorable conditions for failure along mostly its daylighted surfaces. The strength of the discontinuities was estimated based on the result of the Schmidt hammer and found to range from 90 MPa to 140 MPa. As explained earlier, there is a sub-horizontal altered basaltic layer separating two basaltic layers that runs across the slope. The weathered mass in this layer is highly susceptible to erosion in the presence of water, which leads to scouring at the base of the upper layer and, ultimately, failure of the overhanging rocks. There is also another oblique joint intersecting the subparallel one, which creates a favorable condition for rockfall. Similarly, other intersecting fractures play an important role in dislodging rock blocks of varying sizes.

CAUSES OF ROCKFALL

Based on the investigations carried out, the primary causes of rockfall include vertical rock cutting and blasting, inadequate dressing of rock face after blasting, the intersection of the joints exposed on the face of the slope forming wedges, the presence of a prominent oblique joint/fracture between the two separate layers of basalt, a weak subhorizontal layer dividing two basaltic layers, showing on the face of the slope, a smooth joint surface or sheet joint of the top of the slope acting as a sliding surface through which most of the material slides down, a large amount of precipitation with high intensity, seepage of water into the weathered and weak layers that are exposed on the surface after the construction of the expressway, a high gradient of slope and a steep vertical cut.

ROCKFALL PREVENTIVE MEASURES

The vertical face, which is vulnerable to sudden detachment of rocks, was mitigated to prevent dislodged rock from falling off the vertical cliff. The steep vertical cuts with loose overhanging blocks were leveled and the loose rock was removed. Since the rocks have multiple open joints and are likely to widen over time, the application of steel fiber reinforced shotcrete (SFRS) of 70 mm thickness was suggested along with covering the entire sledge with a steel cable net of aperture size (300 mm x 300 mm) in order to stabilize the rock slopes. Steel fibers are very efficient compared to other material fibers such as asbestos, nylon or polypropylene and glass due to their high tensile strength and adherence to concrete. The SFRS provides measures to prevent the rock blocks from moving off the slope and also will prevent the weathering of the rock slope. This kind of stabilization is not intended to prevent the movement of blocks but rather to control rockfall from bouncing onto or blocking the expressway. For installing the steel cable net on the slope, a trench of 30 cm x 50 cm at a distance of 1.5 m from the top of the cliff was excavated. Iron rods were inserted 0.5 m from the bed of the trench. After inserting the rods, the holes were grouted with cement concrete. Subsequently, the steel rope net was laid and the trench was filled with cement concrete for anchorage. The steel cable was spread across the vertical face of the slope up to the toe. The cable net was anchored on the surface of the slope with expansion bolts/grouted bolts of 0.50 m length.

The remedial measures suggested were implemented in 2006 on some of the highway's stretches. In a few locations, the slopes were stabilized with SFRS and steel cable net; at others, the slopes were treated only with steel cable net. The slopes are being physically monitored by the Central Road Research Institute (CRRI) till September 2010. Figures 8 and 9 show the remedial measures that were implemented on the rock slope. The rocks were detached from the sloping surface where the slope is treated with steel cable net. Some rocks are trapped in the net and some were falling at the toe of the slope.

Figure 8. Remedial measures implemented on vertical slope (shotcrete and wire mesh).

Figure 9. Trapped rock blocks at the toe of the slope.

SUBSIDENCE

The subsidence portion (Figure 1) is located at km 41 on the downhill side of the expressway, adjacent to Adoshi tunnel (which extends for 500 m along the expressway). Figure 10 shows the schematic diagram of the subsidence problem on the expressway. The annual rainfall in this area is 200–300 cm, and the present drainage measures are not sufficient to drain this amount of water effectively. During the rainy season, water causes significant damage to this road at four locations in the form of a waterfall. One is in the curved portion, the second one is in between the curve and entrance of the tunnel, the third one is above the tunnel and the fourth one is after the tunnel exit. Pipe culverts were provided to help drain out the water on this stretch of the road, but the gradient of these culverts is not proper. However, it was observed that the internal diameter of the pipes varies from 1 to 1.2 m. In the pipe culvert, two pipe sections 1.0 m in internal diameter are provided, and the remaining pipes are 1.20 m in internal diameter. Two 1.0 m diameter pipe sections are installed in between the 1.20 m pipe sections. The joint between the 1.0 m diameter pipe and the 1.2 m diameter pipe is not proper, and the gap between the two pipes is 30 cm (Figure 11). Remaining pipes are also not joined properly. During periods of heavy rain, water seeps through the joints and saturates the surrounding soil mass, thereby causing cracking between lanes (i.e., flexible and rigid pavement). The runoff from the pipe culvert and cross drains is draining directly onto the embankment surface (Figure 12). The rainwater scours the embankment soil, thereby causing the instability of the embankment. After the exit of the Adoshi tunnel, the gradient of the drain is not proper. As a result, water may stagnate and seep into the pavement through the existing weep holes. The rigid pavement (on the outermost side lane) subsided by 5 to 6 cm, and subsidence also has occurred in between the joint of fast track lane (middle lane) and the truck lane. The average width of a crack is 5 cm and the average length of a crack is 85 m (Figure 13). The surface between the crash barrier and the rigid portion has been paved with bitumen. The flexible portion or shoulder (bituminous surface) has separated from the rigid pavement; the depth of the crack is 50 cm (Figure 14).

A crash barrier has been provided along the slope. Beyond the crash barrier, the average width of the embankment on the downhill extends downhill another 12 m. Drainage holes (Figure 15) are provided in the crash barrier at frequent intervals, but the holes are left open directly on the embankment surface. The considerable quantity of water passing through these drainage holes is causing saturation of the embankment fill beyond the parapet and is weakening the foundation of the crash barrier as well as the scouring below the crash barrier and pavement (Figure 16). A longitudinal drain is provided after a certain distance from the crash barrier on the embankment. From the crash barrier to the longitudinal drain, the surface is not lined and scouring was observed below the crash barrier. The crash barrier is also tilted, and below the crash barrier, the embankment slope has experienced scouring up to a depth of 1 m. The embankment has subsided by 2 to 3 m in steps, and each step has an average height of 1 m (Figure 17).

The height of the embankment ranges from 15 m to 35 m, and the embankment slope varies from 35^0 to 45^0. At the toe of the embankment, a 2-meter high gabion wall was constructed to support the slope. The gabion wall is bulging at some places, which indicates movement of the embankment slope. The gabion wall is also obstructing the flow of water coming from pipe culverts.

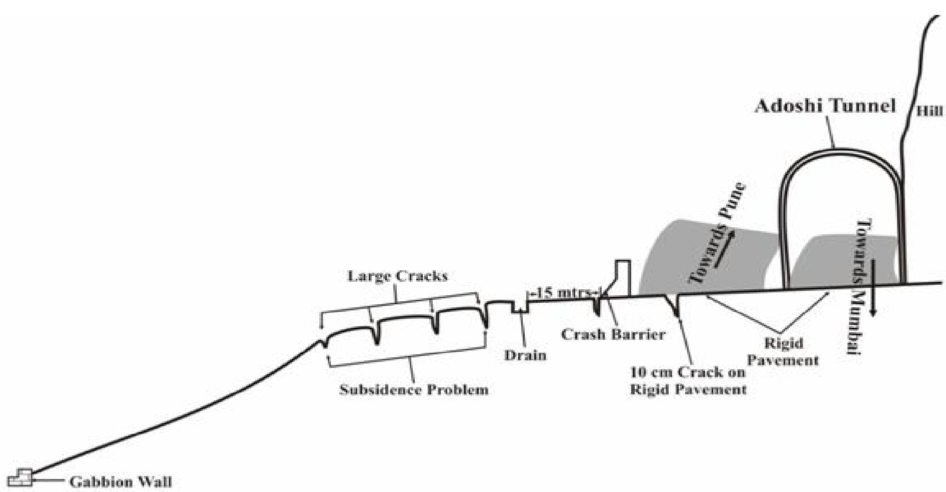


Figure 10. Schematic diagram showing subsidence problem on the Mumbai-Pune expressway.

Figure 11. Dislodged pipe culvert.


Figure 12. Pipe culvert without stepped chute.

Figure 13. Dislodged rigid pavement.

Figure 14. Crack between rigid and bituminous pavement.

Figure 15. Drainage holes left open on embankment surface.

Figure 16. Embankment scoured below the crash barrier.

Figure 17. Subsidence of the embankment.

STABILITY ANALYSIS

Soil samples were collected from the field. In-situ density of the embankment was measured using the core cutter method. The bulk density of the embankment is 17 kN/m^3. The shear strength parameters of the embankment soil are c'=3 kPa and φ'=30^0. The subsidence portion is around 500 m in length. The cross sections A – A, B – B, C – C and D – D were chosen for every 125 m as shown in Figure 18. Stability analysis of the embankment slope was analyzed by using GEO 4 software (Figure 19). The factor of safety of the embankment was analyzed using Bishop's and Peterson's methods. These results are presented in Table 1. The factor of safety at cross sections A – A and B – B is less than 1.25, which is considered critical by both the code of Indian standards and the US Army Corps of Engineers (IRC: 75, 1979 and EM 1110- 2-1902, 2003).

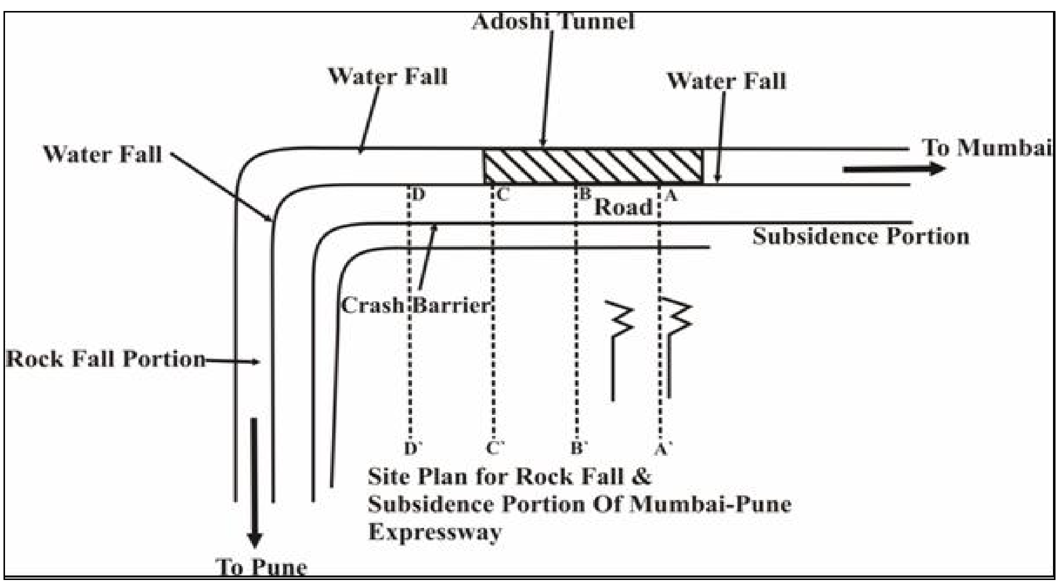


Figure 18. Location of crosss sections for stability analysis.

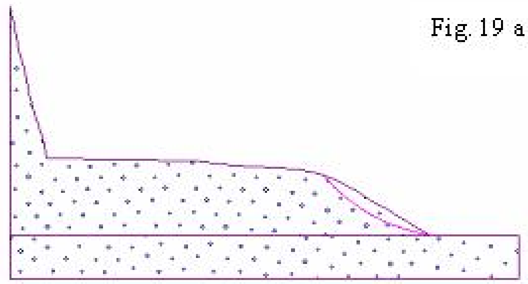
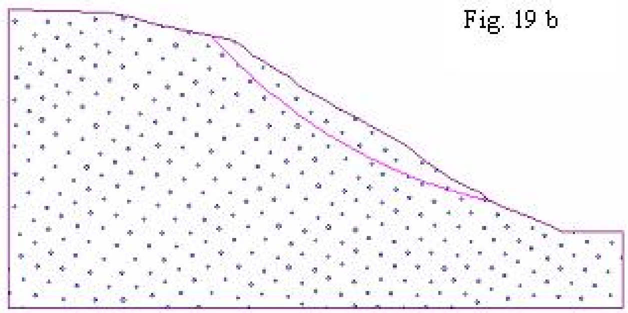
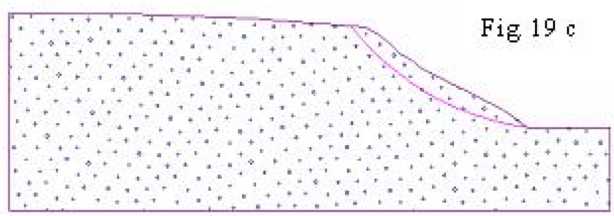
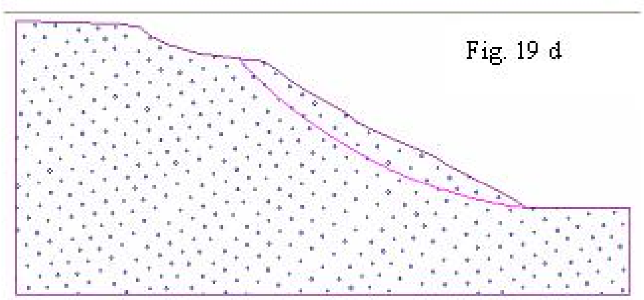
Cross section	Stability analysis	Factor of Safety	
		Bishop method	Peterson Method
A – A	 Fig. 19 a	1.17	1.14
B – B	 Fig. 19 b	1.14	1.11
C – C	 Fig. 19 c	1.38	1.34
D - D	 Fig. 19 d	1.46	1.42

Figure19. Slope stability of subsidence portion.

Table 1: Stability analysis of subsidence portion.

S.No	Cross Section	Factor of Safety	
		Bishop Method	Peterson Method
1	A – A	1.17	1.14
2	B – B	1.14	1.11
3	C – C	1.38	1.34
4	D – D	1.46	1.42

CORRECTIVE MEASURES FOR SUBSIDENCE

The following remedial measures were implemented to avoid further deterioration of the embankment and its slopes:

- The broken culvert was repaired by providing uniform-sized pipe sections, and the joints were properly sealed.
- On the shoulder portion and below the crash barrier, the cracks were sealed with cement grouting and monitored for three years. The portion stabilized and no further cracks were observed in this area.
- After the crash barrier, the embankment was excavated up to 3 m and recompacted to 95% of maximum dry density and moisture content.
- Longitudinal and transverse drains were provided at frequent intervals.
- To prevent erosion and to stabilize the embankment, the slope has been protected with stone pitching/turffing.

CONCLUSIONS

The Mumbai-Pune Expressway was opened to traffic in March of 2002. During the subsequent monsoon period in 2003 and 2004, the expressway experienced rockfall, landslides and subsidence at seventeen critical locations. One of the problematic areas at km 41 on the expressway was chosen for detailed study. The rockfall and subsidence phenomenon has been investigated separately from geological, geomorphological and geotechnical aspects. Based on field observations and investigations, the most probable causes and mechanisms of rockfall (as well as subsidence) have been described. Stability analysis of an existing embankment slope of the subsidence portion was analyzed by using GEO 4 software. The factor of safety of the embankment was analyzed by Bishop's and Peterson's methods, and the results indicated the most critical sections. Suitable schemes of remedial measures were recommended for their prevention. The rockfall preventive measures were not intended to avoid the movement of blocks but to control rockfall from bouncing onto or blocking the expressway. For subsidence, the remedial measures included repairing the embankment, sealing cracks, modifying drainages and more. The remedial measures were implemented by the Ministry of State Road Development Corporation (MSRDC) in 2006. The Central Road Research Institute is monitoring the efficacy of these remedial measures.

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